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GRAPHIC DESIGN OF ALLUVIAL CHANNELS

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WATERWAYS DIVISION

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GRAPHIC DESIGN OF ALLUVIAL CHANNELS

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SYNOPSIS

The determination of channel depth and slope to carry a given unit discharge and sediment load is given in graphical form on the basis of the Einstein bed-load function. Applications of these curves are made to evaluate the effects on river regimen by the diversion of flows, construction of dams, contraction of channels, elimination of river forks, and cut-off of bends. The deficiencies of knowledge in describing the canal flows and in determining the sediment supply to the canal systems still prevent the use of the method in designing irrigation canals.

INTRODUCTION

The design of alluvial channels to carry a given flow is one of the important problems in the field of hydraulic engineering. As early as in the eighteenth century, Chezy developed a formula which relates the depth and slope of the channel to the velocity of flow. As time passed, engineers soon realized that the problem is complicated by the fact that alluvial streams carry not only the water flow, but also sediment of the same type as that which composes the bed. Erosion or deposition will occur, if no proper provisions are made for the sediment flow, and thereby change the characteristics of the channel completely. This paper attempts to show how one can utilize the results of sediment research to develop a more rational approach to the problem of alluvial channel design.

The Problem

The problem of alluvial channel design can be stated as follows: Given the bed material, what should be the width, depth, and slope of the channel to accommodate the given discharge and sediment inflow? The sediment inflow and the discharge can be regarded as conditions imposed on the channel by the watershed and are independent of the channel itself. For a channel in equilibrium, we have two conditions to be satisfied and three variables. This leaves one of the three variables remaining undetermined, according to the present state of knowledge.

It is to be noted that the sediment inflow refers essentially to the bed-material load (EINSTEIN and JOHNSON, 1950). The wash load, in spite of its often being the major part of the sediment load and the deciding factor in the silting of reservoirs and lakes, plays only a minor role in the problem of channel stability. Indirectly, the presence of large quantities of wash load

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may be a determining factor in the use of permeable structures for construction works and regulating measures.

The Design Curves

The design of alluvial channels is greatly facilitated by the use of a graphical presentation first suggested by RAYNAUD (1953). The theories developed by EINSTEIN (1950) in describing the alluvial channel flow and in relating the sediment transport rate to the flow conditions are taken as the basis for the construction of the design curves. For illustrative purposes, the following simplifications are introduced such that the curves are of more general use:

a) bank friction is neglected; and

b) bed material is uniform in size.

It is understood that in the actual design where there are not more than a few particular channels to be considered, one can always take all the local conditions into consideration and thereby construct more reliable curves.

Figs. 1-3 give the design curves for the determination of the slope and depth with a given discharge and sediment inflow (including both the suspended-load and bed-load) for channels of fine sand (D = 0.25 mm), of coarse sand (D = 2.5 mm), and of gravel (D = 25 mm), respectively. The discharge and sediment inflow are expressed in terms of quantities per unit width of the channel, leaving the variable width open. These curves are applicable only up to the bank full stage.

The absence of the width from the design curves is not as detrimental as it first appears. Plans on river improvements usually include a certain amount of bank stabilization work, and the width of the channel can be chosen freely within limits. In nature, river channels may also be confined in canyons, or the banks may be consolidated by the growth of vegetation. On the other hand, LACEY (1939) claimed that the width, p, of a natural channel is related to the discharge, Q, by the following formula:

$$p = 2.67\, Q^{1/2} \qquad (1)$$

LEOPOLD AND MADDOCK (1953), after analyzing many streams in the Great Plains and the Southwest of the United States, have come to the conclusion that (a) the relationship between channel width and discharge as given in eq. (1) is valid only for the comparison of various cross sections at corresponding discharges; (b) the constant of proportionality in eq. (1) is actually, among others, a function of the bed material size; and (c) the channel width varies only with the 0.26 power of the discharge when considering the variation in a particular cross section of a river.

An examination of Figs. 1-3 indicates that for a given slope (or depth), the sediment transport rate increases substantially with a slight increase in depth (or slope). This implies that so far as the design of alluvial channels is concerned, considerable tolerance can be allowed in the determination of the sediment transport rate, a fact which should be welcomed by engineers who may have doubts in their minds concerning the reliability of existing sediment transport theories. Figs. 1-3 also show that an increase in discharge results in a much larger increase in its capacity to transport sediment. This has considerable bearing on the planning of river works, and will be made clear in the examples cited in the next section.

Planning of River Works

In planning river works, the problem is often not how to design a new channel to carry the discharge and sediment load, but rather how to regulate and improve the existing channels such that they will function properly. Any alteration of the channel naturally upsets the existing regimen of the river; and it is very important to know how the channel adjusts its width, depth, and slope to achieve the new equilibrium. A comparison between the existing regimen and the future regimen will then reveal whether or not the plan serves its purposes. In this respect the use of design curves like Figs. 1-3 is of great help. In the discussion to follow, hypothetical examples will be cited to illustrate how different engineering measures affect the characteristics of streams.

1) Diversion of Flow

Flow is diverted from streams either to relieve downstream flooding, or for irrigation and other domestic and industrial uses. A division of flow naturally induces certain changes downstream, and it is important to ascertain whether the changes will lead to adverse consequences. For instance, for some time part of the Mississippi River flow has been diverted to the sea through the Old River-Atchafalaya channel 320 miles upstream from its mouth. The latter channel is enlarging at an alarmingly rapid rate, and currently takes 23% of the Mississippi River flow. A control structure has been planned by the corps of Engineers to be installed in Old River. Unless this is done during the next 20 years, most of the flows will be diverted away from the lower Mississippi, leaving the port of New Orleans more or less isolated. A crisis of this kind arises essentially from an unfavorable division of the sediment load between the two channels, causing aggradation in one channel and degradation in another. The amount of sediment drawn by the branch channel depends very much upon the local conditions, the controlling factors of which have been clearly pointed out by THOMAS (1953). One must either control the division of flow according to the existing conditions, or change the flow patterns at the point of diversion by engineering measures such that the resulting load division will cause neither erosion nor deposition downstream. In either case it is the question of what should be the division of the sediment supply for a given division of flow that will result in equilibrium flow at existing channel slopes. This can best be answered by plotting the sediment rating curves at constant slopes, a few of which are given in Fig. 4 for different channels. The smaller the slope of the curves is, the larger is the percentage of sediment load that must be diverted by the branch channel without upsetting the equilibrium of the main channel downstream. A few numerical examples are also drawn from Figs. 1-3 and given in Table I. The subscripts 1 and 2 refer to the channels upstream and downstream of the division point, respectively. In case I no change in width is assumed to take place as the result of flow diversion, and in case II the width is assumed to change proportional to the 0.26 power of the discharge.

An examination of Fig. 4 and Table I reveals the following:

a) As the slope of the sediment rating curves is smaller than 45 degrees, the downstream channel will not aggrade only when a larger percentage of sediment load is diverted in comparison with the percentage of flow diversion.

b) For the conditions given in Table I, a diversion of 50% and 25% of the flow calls for a diversion of 65-90% and 35-75% of the sediment load, respectively, if the channel width remains constant. These figures for

TABLE I

EFFECT OF FLOW DIVERSIONS

Bed Material Size D mm.	Q_2/Q_1 * %	q_1 cfs/ft.	S	q_{T1} tons/day/ft	Case I, B = const.			Case II, $B \sim Q^{0.26}$		
					q_2 cfs/ft.	q_{T2} tons/day/ft	$\frac{q_{T2}}{q_{T1}}$ ** %	q_2 cfs/ft.	q_{T2} tons/day/ft	$\frac{q_{T2}}{q_{T1}}$ ** %
0.25	50	100	.00015	32	50	4	13.8	60	7	18.3
			.00025	195		30	15.4		45	19.3
			.00050	1400		260	18.6		400	23.8
		250	.00015	430	125	60	14.0	150	100	19.4
			.00025	2200		340	15.4		600	22.8
			.00050	12000		2400	20.0		4000	27.8
	75	100	.00015	32	75	14	42.8	80	16	46.4
			.00025	195		88	45.1		100	47.5
			.00050	1400		700	50.0		800	53.0
		250	.00015	430	187.5	190	44.2	200	250	53.9
			.00025	2200		1000	45.5		1300	54.8
			.00050	12000		6200	51.8		7000	54.2
2.5	50	100	.0010	75	50	17	22.0	60	25	27.8
			.0015	180		54	30.0		80	37.1
			.0025	495		168	33.9		250	42.1
		250	.0010	340	125	110	32.3	150	150	36.8
			.0015	740		250	33.8		350	39.5
			.0025	2000		690	34.5		1000	41.8
	75	100	.0010	75	75	41	54.7	80	45	55.8
			.0015	180		110	61.1		130	67.0
			.0025	495		315	63.6		375	70.4
		250	.0010	340	187.5	216	63.5	200	230	62.8
			.0015	740		475	64.2		500	62.7
			.0025	2000		1270	63.5		1350	62.6
25	50	100	.003	13	50	-	-	60	1	5.2
			.005	175		13	7.2		35	16.7
			.010	1350		285	21.1		450	27.8
		250	.003	270	125	30	11.1	150	50	15.5
			.005	1350		305	22.6		475	29.4
			.010	5700		2000	35.1		3000	44.0
	75	100	.003	13	75	3	22.0	80	5	35.7
			.005	175		70	40.0		100	53.0
			.010	1350		730	54.0		800	55.0
		250	.003	270	187.5	118	43.7	200	140	48.1
			.005	1350		760	56.3		850	58.5
			.010	5700		3800	66.7		4500	73.2

* Q_2/Q_1 is the percentage of the original flow which enters the downstream channel. $(1 - Q_2/Q_1)$ is the percentage of flow which is diverted.

** Q_{T2}/Q_{T1} is the percentage of the original bed-material load that is allowed to enter the downstream channel without changing the channel slope.

the required diversion of sediment load are reduced to 60-85% and 30-55% if the channel width changes also with the discharge to the 0.26 power.

c) The percentage of sediment load that must be diverted to satisfy the original equilibrium conditions of the channel (i) decreases with the slope and the discharge for the same bed material size; (ii) increases with the bed material size for small streams which carry little sediment load; and (iii) decreases with the bed material size for large streams with heavy sediment load.

2) Construction of Dams

The changes in river regimen caused by the construction of dams across the river have received considerable attention in recent years. The

completion of a number of large dams, including the Hoover and the Elephant Butte Dam, also offers a rare opportunity for close field observations. As a result, it has become common knowledge that damming up the flow will induce aggradation upstream and degradation downstream, two of the immediate effects which can also be derived easily from Figs. 1-3. However, it becomes less obvious when comes the question of what will be the changes in the river regimen below the junction of a sizable tributary downstream from the dam. The elimination of peak flows by the regulation of the reservoir greatly reduces the capacity of the channel to transport sediment. In fact, if the total annual discharge is released from the reservoir at a constant rate, the downstream channel may carry less than one-tenth of the sediment load that the same total annual discharge carries if it flows according to its natural pattern. Or, in other words, the building of a dam upstream reduces both the sediment capacity and the sediment supply in the channel below the junction point, the net effect of which can only be determined by examining the design curves.

Consider, for example, the junction of the two channels as shown in Fig. 5. Will the channel below the junction degrade or aggrade if the discharge from the main channel is cut to 20,000 cfs by the regulation of a reservoir? Table II(a) gives the characteristics of these three channels before the construction of the dam in channel (1). It can be seen that the slope of the channel below the junction is lower than that of either of the confluents, a fact substantiated by many field observations (MACKIN, 1948). The slope of the main channel below the confluence with a sizable tributary will be steeper than that above the junction only when (a) the tributary brings in sediment substantially coarser than that carried by the main stream, such as the increase in Missouri River slope below the mouth of the Platte River; or (b) the tributary carries a disproportionately large sediment load with respect to its discharge, such as the steepening of the Rio Grande River below the confluence of Rio Puerco River. Table II(b) indicates the change in channel characteristics after the completion of a dam in channel (1). Two cases are studied. In case I the degradation below the dam has not yet been extended to the reach immediately above the junction such that the slope of the channel in that reach remains unchanged. In the second case, the degradation below the dam is completed and channel (1) becomes essentially non-alluvial in nature and carries practically no sediment. In both cases the channel below the junction is substantially accreted. This is true also for channels other than these cited in the example above. It appears that the effect of reduction in peak discharge by the operation of the reservoir by far outweighs the effect of decrease in sediment supply, and one may regard the aggradation of the main stem downstream of the dam and below the junctions with tributaries as a rule common to many alluvial streams.

3) Contraction of Channel

One of the most common measures in increasing the navigable depth of a river is to confine the channel between jetties or training walls for limited or extensive stretches. A reduction in width means a proportionate increase in unit discharge and unit sediment supply, the effect of which on the channel characteristics can be derived from the design curves. Table III shows the changes in slope and water depth if the channel is contracted by 50%. Generally speaking, this results in an increase in depth of 45-80% and a decline in slope of 15-30%. STRAUB (1934) has developed a formula expressing the depth in the contracted

TABLE II(a)

Channel Characteristics at the Junction of a Sizable Tributary
(Before the Construction of Dam in the Upstream Main Channel)

Channel	Discharge Q cfs	Sediment Load tons/day	Width[2] p ft.	Unit Discharge cfs/ft.	Unit Sediment Load tons/day /ft	D=0.25 mm			D=2.5 mm			D = 25 mm		
						Slope S	Depth d ft	$\frac{S_3-S_1}{S_1}$[1] %	Slope S	Depth d ft	$\frac{S_3-S_1}{S_1}$[1] %	Slope S	Depth d ft	$\frac{S_3-S_1}{S_1}$[1] %
1	100,000	20,000	1,000	100	20	.00014	18.0	-	.00068	16.8	-	.00325	13.5	-
2	25,000	5,000[3]	500	50	10	.00020	10.7	-	.00089	10.4	-	.00477	7.6	-
3	125,000	25,000	1,120	112	22	.00013	20.0	-7.15	.00063	18.5	-7.35	.00308	14.8	-5.23

(1) Change in slope of the main stream after the junction with a sizable tributary.
(2) Width $\sim Q^{\frac{1}{2}}$
(3) The sediment load carried by channel 2 is so assumed that the following relationship obtained by Leopold and Maddock (1953) is observed:

$$d_2 = (Q_2/Q_1)^{0.4} d_1$$

TABLE II(b)

Channel Characteristics at the Junction of a Sizable Tributary
(After the Construction of Dam in the Upat ream Main Channel)

Channel	Case	Discharge Q' cfs	Width p' ft	Unit Discharge cfs/ft	D = 0.25 mm					D = 2.5 mm					D = 25 mm				
					Total Sediment Load tons/day	Unit Sediment Load tons/day /ft	Slope S'	Depth d' ft	$\frac{S_3'-S_3}{S_3}$[1] %	Total Sediment Load tons/day	Unit Sediment Load tons/day /ft	Slope S'	Depth d' ft	$\frac{S_3'-S_3}{S_3}$[1] %	Total Sediment Load tons/day	Unit Sediment Load tons/day /ft	Slope S'	Depth d' ft	$\frac{S_3'-S_3}{S_3}$[1] %
1	1	20,000	658[2]	30.4	658	1.0	.00014	12.5	-	526	0.8	.00068	10.2	-	66	0.1	.00325	7.5	-
	2				0	0	-	-	-	0	0	-	-	-	0	0	-	-	-
2	1 & 2	25,000	500	50.0	5,000	10.0	.00020	10.7	-	5,000	10.0	.00089	10.4	-	5,000	10.0	.00477	7.5	-
3	1	45,000	858[2]	52.5	5,658	6.5	.00016	13.5	23.1	5,526	6.5	.00079	11.6	25.4	5,066	5.9	.00416	8.4	35.1
	2				5,000	5.8	.00155	14.0	19.2	5,000	5.8	.00078	11.7	23.8	5,000	5.8	.00416	8.4	35.1

(1) Change in slope below the junction after the completion of a Dam in the upstream main channel.
(2) $p_1' = (Q_1'/Q_1)^{0.26} p_1$ and $p_3' = (Q_3'/Q_3)^{0.26} p_3$

TABLE III

Effect on the Channel Characteristics
by Contracting the Channel by 50%

D mm	S_1	q_1 cfs/ft.	q_{T1} tons/day /ft	d_1 ft	q_2 cfs/ft.	q_{T2} tons/day /ft	S_2	d_2 ft	$\frac{S_2-S_1}{S_1}$ %	$\frac{d_2-d_1}{d_1}$ %
0.25	.002	12.5	0.4	6.75	25.0	0.8	.000157	10.5	-21.5	55.6
		25.0	2.5	8.80	50.0	5.8	.000150	13.9	-25.0	58.0
		50.0	12.5	10.75	100.0	25.0	.000145	17.4	-27.5	61.8
		100.0	80.0	13.30	200.0	160.0	.000140	21.0	-30.0	57.9
	.004	5.0	0.5	2.90	10.0	0.9	.000300	4.6	-25.0	60.0
		12.5	4.0	3.90	25.0	8.0	.000300	6.2	-25.0	59.0
		25.0	25.0	5.20	50.0	50.0	.000310	7.7	-22.5	48.1
		50.0	125.0	6.50	100.0	250.0	.000285	10.6	-28.7	63.1
		100.0	600.0	8.95	200.0	1200.0	.000230	16.5	-42.5	84.3
		200.0	4000.0	13.30	400.0	8000.0	.000255	23.6	-36.2	77.5
2.5	.002	12.5	4.0	3.25	25.0	8.0	.0014	5.7	-30.0	74.5
		25.0	25.0	4.50	50.0	50.0	.0015	7.5	-25.0	66.7
		50.0	125.0	6.20	100.0	250.0	.0017	9.2	-15.0	48.3
		100.0	350.0	8.45	200.0	700.0	.00174	11.8	-13.0	41.3
		200.0	900.0	11.00	400.0	1800.0	.00165	17.2	-17.5	56.3
	.001	12.5	0.1	5.40	25.0	0.2	.00073	8.6	-27.0	58.4
		25.0	2.0	7.00	50.0	4.0	.00072	12.0	-28.0	71.5
		50.0	15.0	9.65	100.0	30.0	.00077	15.4	-23.0	59.6
		100.0	65.0	12.80	200.0	130.0	.00083	18.5	-17.0	44.5
		200.0	180.0	16.00	400.0	360.0	.00075	26.0	-25.0	62.5
25	.003	50.0	0.2	9.90	100.0	0.4	.0020	18.0	-33.3	81.8
		100.0	12.5	14.20	200.0	25.0	.0022	23.1	-26.6	62.7
		200.0	140.0	19.30	400.0	280.0	.0023	34.0	-23.3	76.2
	.006	25.0	1.5	4.70	50.0	3.0	.0042	8.1	-30.0	72.3
		50.0	40.0	6.60	100.0	80.0	.0041	11.7	-31.6	77.3
		100.0	300.0	9.30	200.0	600.0	.0047	14.9	-21.6	60.2
		200.0	1300.0	12.90	400.0	2600.0	.0047	20.8	-21.6	61.2

section in terms of the flow conditions in the uncontracted section as follows:

$$d_2 = \frac{\frac{d_1}{(1-\alpha)^{5/7}}\left\{\frac{-T_0}{\gamma S_1} + \left[\left(\frac{T_0}{\gamma S_1}\right)^2 + \frac{4d_1\left(d_1 - \frac{T_0}{\gamma S_1}\right)}{1-\alpha}\right]^{1/2}\right\}^{5/7}}{\left[2\left(d_1 - \frac{T_0}{\gamma S_1}\right)\right]^{5/7}} \qquad (2)$$

in which d_2 = water depth at the contracted section,
d_1 = water depth at the un-contracted section,
T_0 = critical tractive force,
S_1 = slope in the un-contracted section,
γ = unit weight of the water,
α = the amount of contraction
= $(p_1 - p_2)/p_1$, p_1 and p_2 being the widths of the channel at the normal and contracted sections, respectively.

For most of the streams, the critical tractive force is small and can be ignored. Eq. (2) thus becomes

$$d_2 = d_1/(1-\alpha)^{9/14} \qquad (3)$$

and

$$(d_2 - d_1)/d_1 = 1/(1 - \alpha)^{9/14} - 1 \qquad (4)$$

α equals 0.5 for 50% reduction of channel width; and eq. (4) gives an increase in depth of 56%, a value which checks well with the results given in Table III.

4) Closure of River Forks

When a river flows in a divided channel, the water depth in either of the channels is often insufficient for navigation at low water, and it is desirable to eliminate the forks. Consider, for instance, half of the flow goes down each arm as shown in Fig. 6: Table IV shows the channel characteristics of the river at the fork and in the undivided reach. The water depth in the forks is only 80% of the depth in the single course if the flow is equally divided at the fork. It is interesting to note that FRANZIUS (1936) gives the ratio of d_2/d_1 from 0.777 to 0.80 by making several assumptions concerning the hydraulics of the channel.

TABLE IV

Channel Characteristics of a River
At the Fork and in the Undivided Reach

D mm	Undivided Channel		Channels at the fork		d_2/d_1
	d_1 ft	S_1	d_2 ft	S_2	
0.25	12.7	.000214	10.3	.000250	0.81
2.5	11.8	.00114	9.7	.00126	0.82
25	11.3	.00430	8.8	.00500	0.78

5) Cut-Off

When a stream flows in excessive meander loops, the water at high stages may take a short-cut across the narrow necks of overdeveloped bends. This natural process is often assisted by engineering measures so as to relieve the flood pressure upstream and, to a lesser degree, to increase the maneuverability of barge fleets. A cut-off of the bends means a shortening of the river course, and consequently, a local steepening of the channel. This local increase in slope will eventually be wiped out by lowering of bed upstream and by a raise of river bottom downstream. If the channel is subject to slow accretion, the step will be absorbed in the final adjustment of the profile, reducing the aggradation of the upstream channel to an amount equal to the product of the length of the cut-off channel and the final equilibrium slope. The reliance on cut-offs in reducing flood stages upstream, therefore, is only effective when either the channel is slowly building up at the present, or the downstream aggradation causes no serious local damages.

It has been demonstrated so far that valuable information on the interdependent effects of changes of the channel characteristics, of the sediment supply, and of the flow can be derived from proper use of the design curves. In addition to this, there are other complications which must be considered so as to obtain a general picture as complete as possible. Any change in conditions may lead to other changes, and thereby further complicate the final adjustment. For instance, lowering of the channel bed may increase the sediment inflow from tributaries resulting from the increase in slopes near their mouths (LANE, 1942). There are also conditions which are

peculiar to the particular stream under consideration. No two natural streams are exactly alike. Each has had its own complex history, and possesses certain features and irregularities not to be found in other similar drainage basins. For instance, the existence of gravel lenses or outcrop of rock in the bed may allow the bed to be scoured only down to a certain extent, beyond which the flow is forced to widen the channel which may not be the case otherwise. An adjustment of the slope may be accomplished by the difference in channel filling or scouring between the upstream and downstream part of the reach, or may be achieved by the change in meander pattern, depending very much on the erodibility of the bank material. Rubey (1952) has also pointed out that a change in regimen is liable to be counter-acted by an adjustment in cross-section in some cases, and by modification in slope in others, depending on the historical background and physiographical and geological features of the watershed.

The design curves also do not give any clue on the time factor involved in planning the river works. Besides the outcome of the regulating measures, it is just as important to know how long the river takes to reach its new equilibrium in response to the changes. A plan which will bring immediate relief, but in the long run will retreat to its old conditions, may still prove to be economically feasible if the time involved between the immediate change and the final outcome is sufficiently long.

Design of Irrigation Canals

The design curves as given in Figs. 1-3 cannot be used for irrigation canals design because of the lack of certain information. The difficulties lie partially in the description of the hydraulics and partially in the determination of the sediment supply.

Irrigation canals are usually of much smaller size than natural streams. The bank friction is a significant part of the total resistance. The amount of bank friction depends very much on the shape of the channel, thus making the construction of generalized design curves impossible. Lacking are also systematic studies on the factors which govern the additional resistance caused by the existence of sand bars and other irregularities at the bed surface of narrow channels. It has been found for natural streams that the bar resistance is a function of bed-load transport (EINSTEIN and BARBAROSSA, 1952), and the functional relationship between the two has been incorporated in the preparation of Figs. 1-3. For more confined canal flows, this empirical relationship between the bed-load transport and the bar resistance appears to be somewhat different. There are also indications that other factors, such as the width of the channel, may also come into the play. This deficiency of knowledge calls for prototype measurements which must be analyzed in the light of modern concepts on friction in open channel flow (EINSTEIN and BANKS, 1951).

The determination of sediment inflow in canals is rather complicated. In planning river works, the sediment supply to the reach under consideration can be calculated by applying the Einstein bed-load function to the reach immediately upstream. The sediment inflow to the canal system, however, depends not only on the sediment load of the stream from which the water is diverted, but also on the division of sediment load as the result of flow diversion. The latter differs considerably according to the local conditions; and the data available is so meager that not even a rule of thumb can be made at the present. This situation may be improved considerably if data on the

division of sediment load under a variety of flow conditions can be collected either by taking direct field measurements of sediment load or by applying the Einstein bed-load function to both the stream and the canal, after the latter has become stable and provided the bar resistance of canal flows has been properly described. In many irrigation systems, provisions have been made at or close to the canal entrance to eject the coarse material from the canal. Today little is known as how effective are these ejectors. In case they do exclude all the coarse sediment, the material drawn from the stream to the canal will consist mainly of the fine particles. The distribution of fine sediment in the stream is generally more uniform, and the division of these particles is probably in proportion to the division of discharge at the canal entrance. This condition permits the determination of sediment inflow to the canal system with ease. Unfortunately there is one more complication. The canal inflow no doubt carries with it its share of the wash-load material. Since the velocity in the canal is lower than that in the stream, part of the wash load may settle out in the canal and form a significant part of the bed. Or, in other words, these fine particles which behave like wash-load in the stream may behave like bed-material load in the canal, and their transport rate must also be included in the stability consideration. How much of this part of the load is diverted depends not only on the flow, but also on the availability of the material from the upstream watershed. For a small watershed it may be possible to establish a rating curve of the wash load material by tedious sediment load sampling program. This task becomes increasingly complex if the watershed is large and if the topography, soil, and vegetation differ considerably from place to place within the watershed and if the local rainfall patterns cover only part of the entire area. On the other hand, it is to be remembered that one does not need to know the sediment inflow very accurately, and perhaps complications like these can be ignored with little consequences. No experience exists on this part, however.

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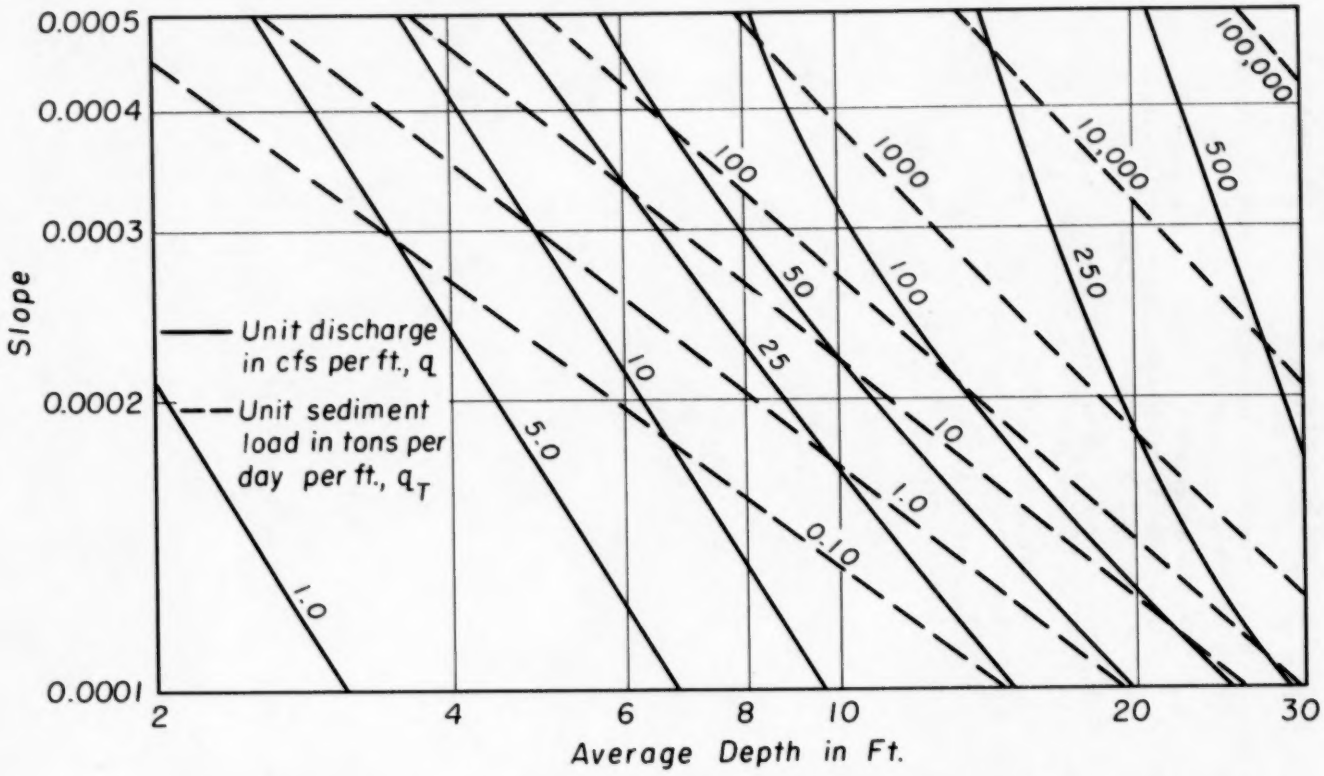


Fig. 1 Design of Alluvial Channels to Carry the Given Discharge and Sediment Load Fine Sand Bed (D = 0.25 mm.)

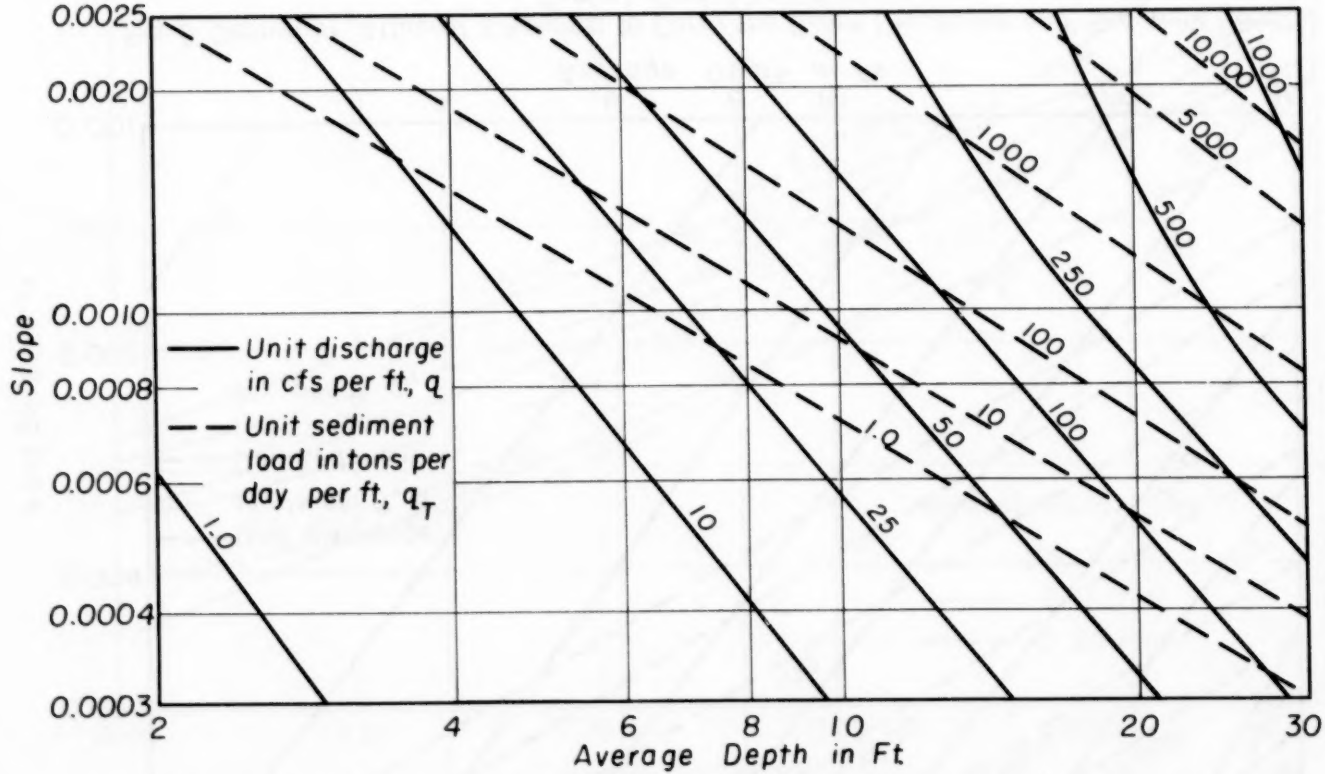


Fig.2 Design of Alluvial Channels to Carry the Given Discharge and Sediment Load Coarse Sand Bed (D = 2.5 mm.)

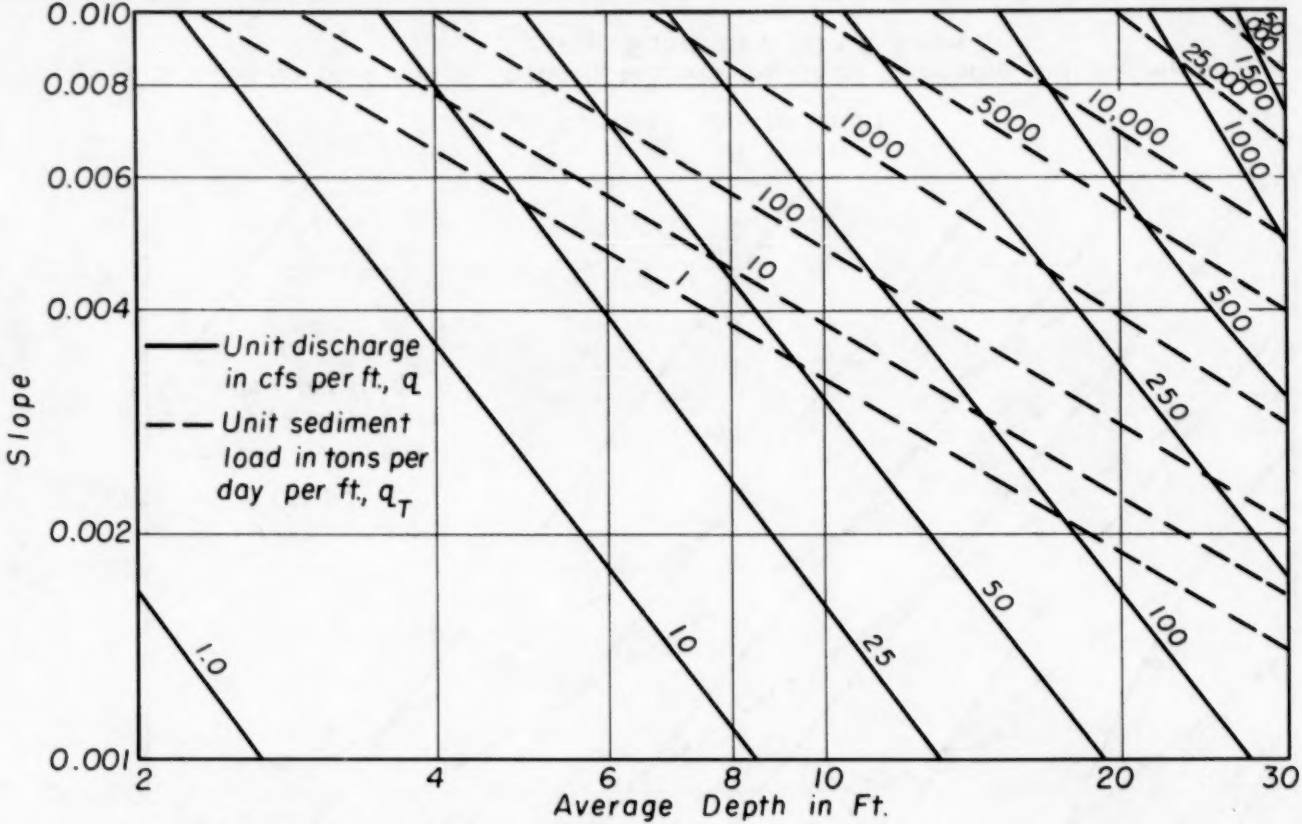


Fig. 3 Design of Alluvial Channels to Carry the Given Discharge and Sediment Load Gravel Bed (D = 25 mm.)

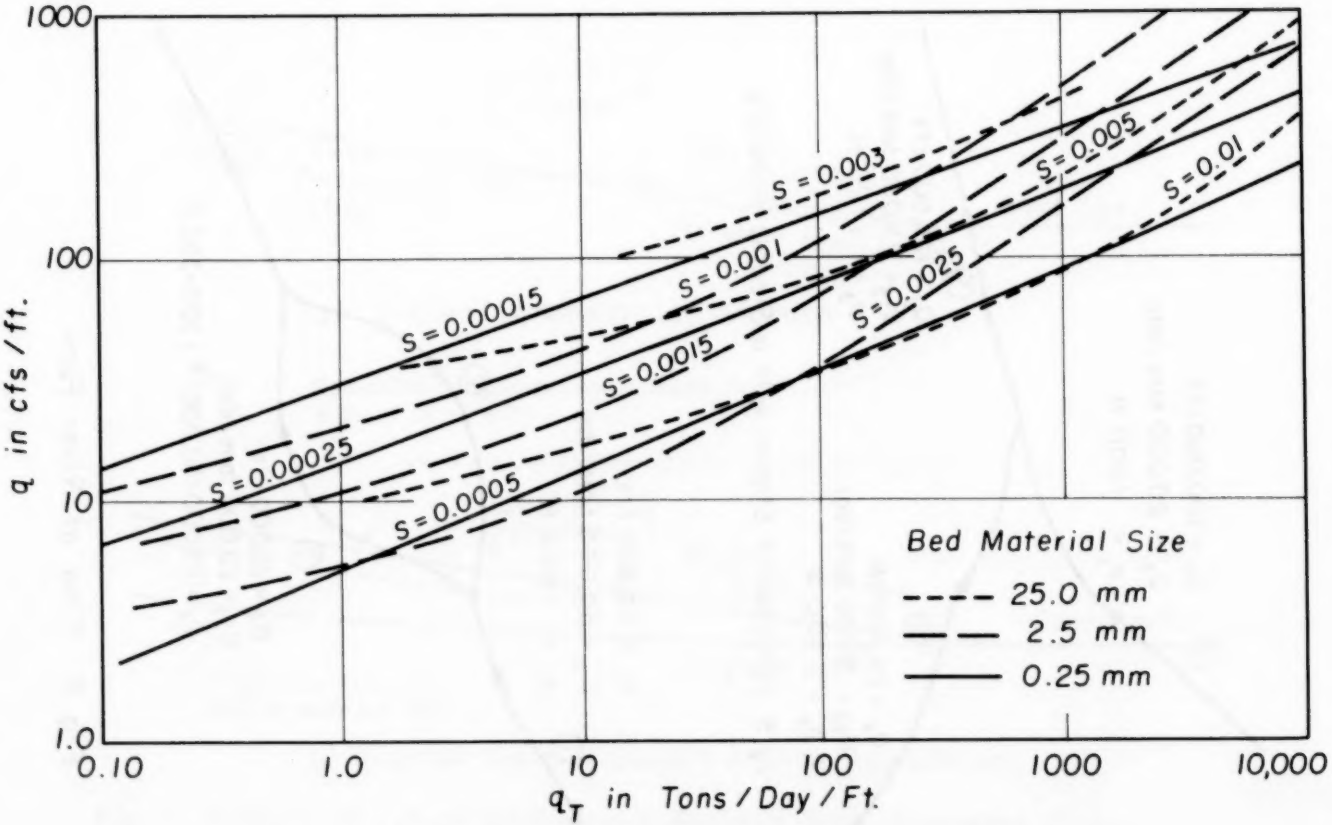


Fig. 4 Sediment Rating Curves at Constant Slopes

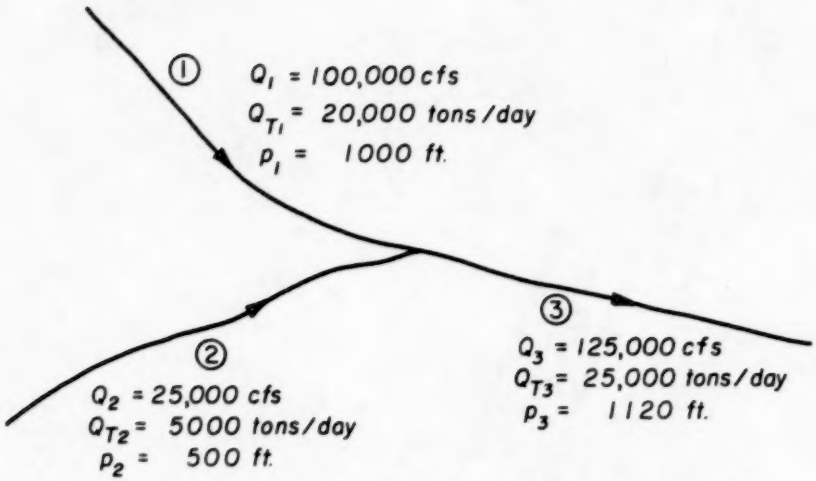


Fig. 5 Junction of Stream with a Sizable Tributary

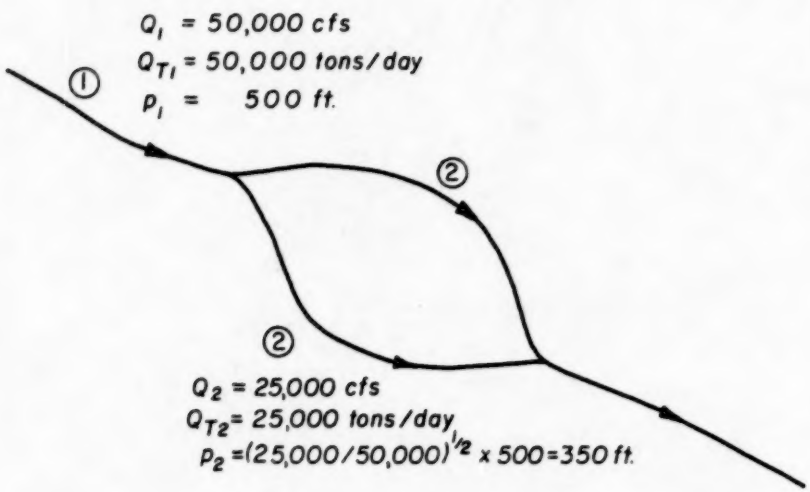
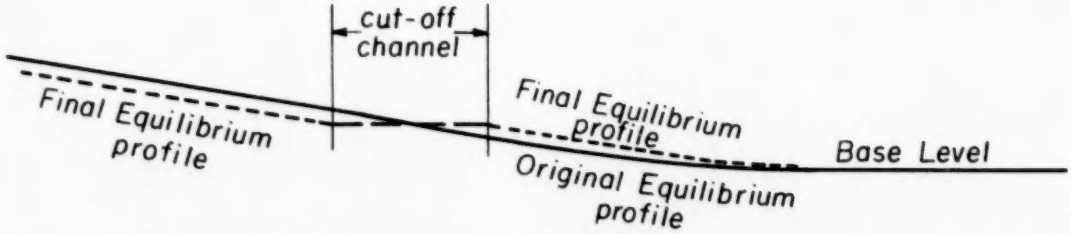
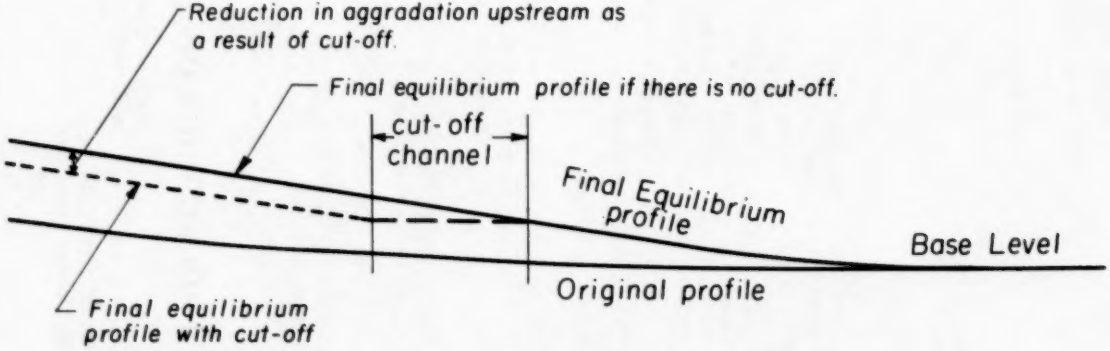


Fig. 6 Flow at River Fork



(a) Original Channel in Regimen



(b) Original Channel Subject to Slow Aggradation

Fig. 7 Effects of cut-off on Rivers in Regimen & on Aggrading Rivers.

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